UNIT 7 INTRODUCTION TO MASONRY

Structure

7.1 INTRODUCTION

Though most of the masonry structures or their elements, before the first half of twentieth century, used to be constructed on thumb rules based on experience; it is only after the advent of IS:1905-1961, Code of Practice for Structural Use of Unreinforced Masonry, 'Calculated or Engineering Masonry' have been in use. This code was subsequently revised in 1969 and in 1987 based on extensive research and expertise developed during these years.

Masonry is an assemblage of masonry units (bricks or blocks) bonded together with mortar.

In this Unit, (a) design aspects of masonry made of common burnt clay bricks and cement-sand mortar and (b) structural design of building elements (such as walls, pillars and footings), and bridge elements (such as abutments and piers) have been discussed and illustrated with examples.

Objectives

After going through this Unit, a student will be able to learn

- design specifications and their application for design of walls, pillars and footings of buildings, and
- design specifications and their application for design of piers and abutments of bridges using unreinforced masonry made of bricks and cement-sand mortar.

7.2 DESIGN CONSIDERATIONS OF MASONRY WALLS

From structural design considerations, walls can be classified into two types-load bearing and non load bearing.

7.2.1 Load Bearing Walls

Masonry buildings are mainly constructed of load bearing walls, i.e. where walls are used to transfer gravity as well as lateral loads to the foundation in addition to its common function of subdividing space, providing thermal and accoustic insulation, affording fire resistance and providing weather protection. While transferring design load, the masonry shall have compressive, tensile and shear stresses within permissible limits and the wall will not buckle or overturn.

Design Loads

Design loads to be taken into considerations are :

(a) Gravity loads

  (i) Dead loads - self weight and weight of super-structure,

  (ii) Live load of super structure, and

(b) Lateral loads due to accidental horizontal loads, wind pressure and earthquake.

Permissible Stresses

Permissible Compressive Stress

The **permissible compressive stress** (σ_{cc}) is a modified value of *basic* compressive stress σ_{ccb} since the latter is influenced by slenderness ratio of wall, eccentricity of loading, area of cross section of wall, shape of masonry units and type of loading (uniform or concentrated).

The *basic* permissible compressive stress (σ_{ccb}) of masonry under normal conditions is a function of strengths of its constituentsbricks and cement-sand mortars. The average compressive strength of bricks, mortars and basic compressive strength of masonry are given in Tables 7.1, 7.2, and 7.3 respectively.

Table 7.1 : Average Compresssive Strength of Common Burnt Clay Bricks

Class Designation	Average Compressive Strength Not less than (N/mm^2)	Average Compressive Strength Less than (N/mm^2)
350	35	40
300	30	35
250	25	30
200	20	25
175	17.5	20
150	15	17.5
125	12.5	15
100	10	12.5
75	7.5	10
50	5	7.5
35	3.5	5

Table 7.2 : Mix Proportions and Strength of Mortars for Masonry

Grade of Mortars	Mix Proportions (By Loose Volume)			Minimum Compressive Strength at 28 days
	Cement	Lime	Sand	(N/mm^2)
H1	1	$\frac{1}{4}$ C or B	3	10.0
H2	1	$\frac{1}{4}$ C or B	4	7.5
	1	$\frac{1}{2}$ C or B	$4\frac{1}{2}$	6.0
M1	1	-	5	5.0
M2	1	-	6	3.0
M3	1	-	7	1.5
L1	1	-	8	0.7

Note 1 : Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced in order to achieve the minimum specified strength.

Note 2 : Addition of Lime in H1 & H2 are not essential. Lime in these two mixes are added only to increase workability.

Note 3 : B and C denote semi-hydraulic lime and fat lime respectively.

Table 7.3 : *Basic* Compressive Strength of Masonry (after 28 days)

Mortor Type (Refer Table 7.2)	Basic Compressive Stresses (N/mm^2) Corresponidng to Masonry Units of which Height to width Ratio does not exceed 0.75 and crushing strength (N/mm^2) is not less than											
	3.50	5.0	7.5	10	12.5	15	17.5	20	25	30	35	40
H_1	0.35	0.50	0.75	1.00	1.16	1.31	1.45	1.59	1.91	2.21	2.5	3.05
H_2	0.35	0.50	0.74	0.96	1.09	1.19	1.30	1.41	1.62	1.85	2.1	2.5
M_1	0.35	0.50	0.74	0.96	1.06	1.13	1.20	1.27	1.47	1.69	1.9	2.2
M_2	0.35	0.44	0.59	0.81	0.94	1.03	1.10	1.17	1.34	1.51	1.65	1.9
M_3	0.25	0.41	0.56	0.75	0.87	0.95	1.02	1.10	1.25	1.41	1.55	1.78
L_1	0.25	0.36	0.53	0.67	0.76	0.83	0.90	0.97	1.11	1.26	1.4	1.06

Note 1 : This table is valid for slenderness ratio upto 6 and loading with zero eccentricity.

Table 7.4: Stiffening Coefficients for Walls Stiffened by Piers, Buttresses or Cross Walls

Ratio $\frac{S_P}{w_P}$	Stiffening Coefficient for		
	$\frac{t_P}{t_w} = 1$	$\frac{t_P}{t_w} = 2$	$\frac{t_P}{t_w} = 3$ or more
6	1.0	1.4	1.2
8	1.0	1.3	1.7
10	1.0	1.2	1.4
15	1.0	1.1	1.2
20 or more	1.0	1.0	1.0

where

S_p = centre-to-centre spacing of the piers or cross walls,

t_p = the thickness of pier,

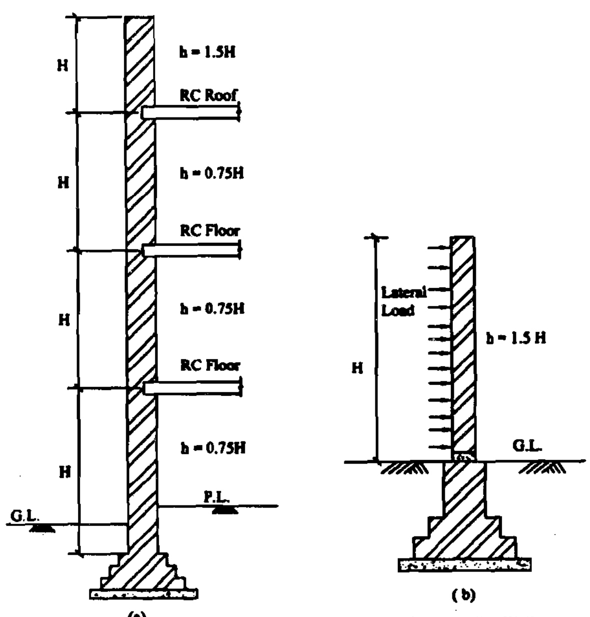
t_w = actual thickness of the wall proper, and

w_p = width of the pier in the direction of the wall or the actual thickness of the cross wall.

Slenderness Ratio (SR)

Slenderness ratio is the ratio of effective height or length of wall to effective width. Both the values one w.r.t. effective height and another w.r.t. length of slenderness ratio shall be caluclated and the *lesser* of the two will be taken for design purposes. For load bearing walls this ratio shall not exceed 27.

Effective height (h) of a wall supporting RCC floor/roof and that of free standing wall are shown in Figure 7.1.



(a) Effective Height of Wall Supporting RC Floor/Roof irrespective of Direction of Span

(b) A Free Standing Wall

Figure 7.1 : Load Bearing Walls

Effective Length (*l*) of wall shall be defined as diagramatically represented in Figure 7.2. When slenderness ratio is based on its effective length, a cross wall/ piers/buttress shall be deemed to provide effective lateral support if it has the minimum dimension shown in Figure 7.3.

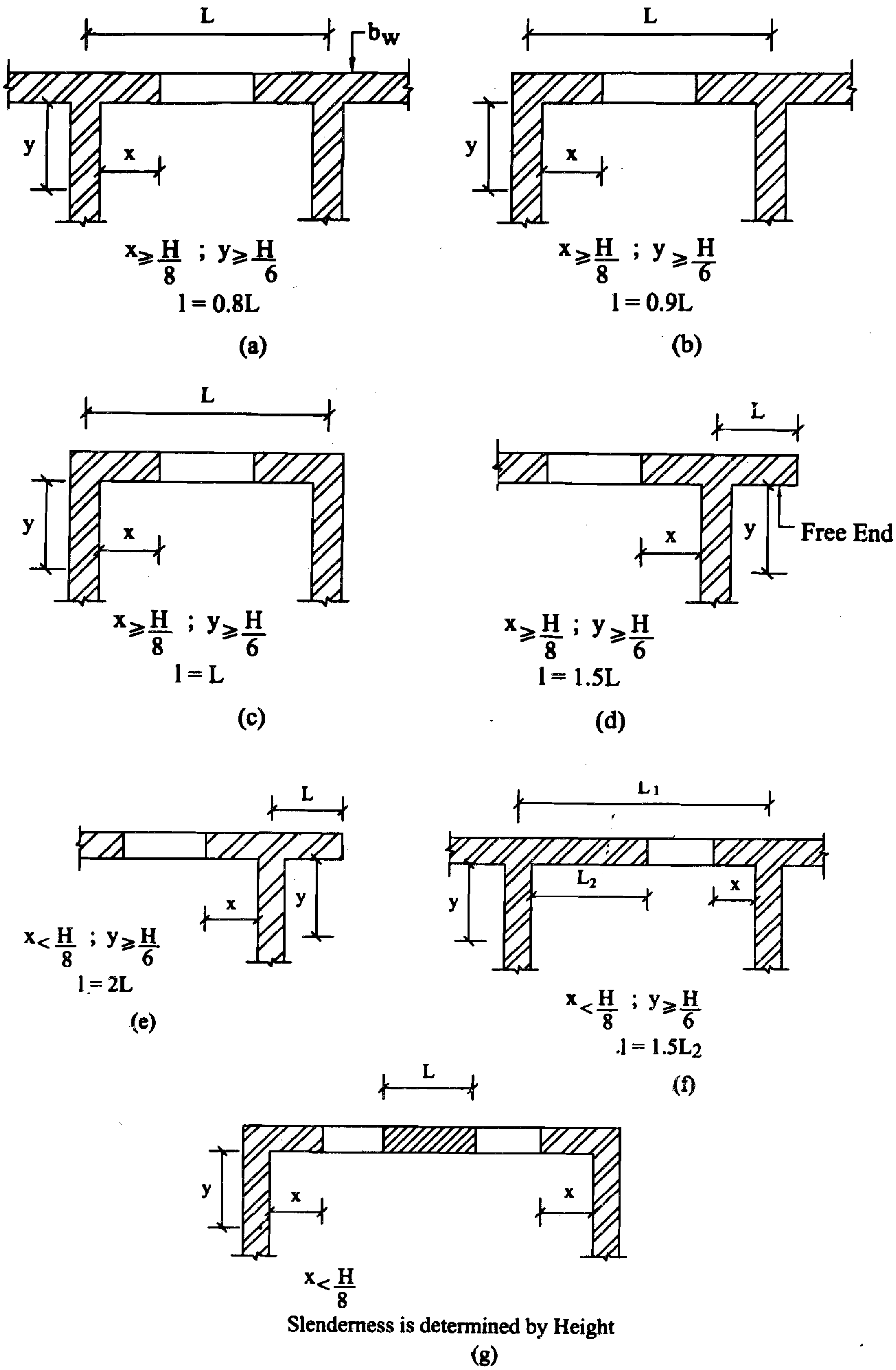


Figure 7.2 : Effective Length of a Wall

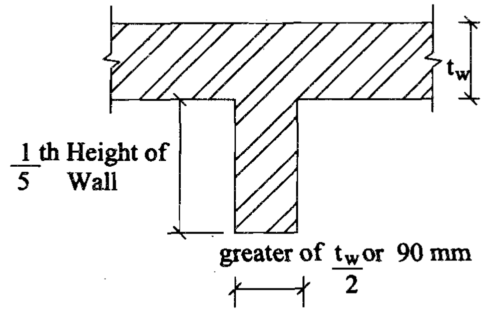


Figure 7.3 : Minimum Dimension for Masonry Wall / Buttress / Pier Providing Effective Lateral Support

Effective thickness (t) of a solid wall shall be actual* thickness of a wall in case of solid wall bonded into pier/buttress. Effective thickness, for determining slenderness ratio based on *effective height only*, shall be actual thickness multiplied by stiffening coefficient as given in Table 7.4.

For solid walls stiffened by cross walls, the cross walls shall be assumed to be equivalent to piers of width equal to the thickness of the cross wall and of thickness equal to three times the thickness of stiffened wall for determining stiffening coefficient from Table 7.4.

Example 7.1

Determine slenderness ratio of wall A of first floor of Example 7.3 (Figure 7.8) having window size 2m×1.6m.

Solution

Effective Height (h)

Though length of wall A is less than 4 times its thickness it will not be treated as a column, since it is supported by cross wall.

Height, H of the wall i.e. c/c distance between 1st and 2nd floors.
= 3 + 0.125 = 3.125 m (vide Figure 7.1) and correspondingly
h = 0.75 H = 0.75 × 3.125 = 2.34 m

Effective Length (l)

L = 0.35. Having window taller than 0.5 H, this wall is free on one end and supported by cross wall at the other,

$l = 2\text{ L} = 2 \times 0.35\text{m} = 0.7\text{m}$

Effective Thickness (t)

t = 220

$$\text{For h, SR} = \frac{h}{t} = \frac{2.34}{0.22} = 10.64$$

$$\text{and for } l\text{, SR} = \frac{0.7}{0.22} = 3.18$$

∴ Design SR is *lesser* of the above two values i.e. **Design SR = 3.18 Ans.**

* Actual thickness of a wall is computed as the sum of dimension of masonry units specified in relevant standard together with the specified joint thickness. In case of raked joints, thickness shall be reduced by the depth of raking of joints for plastering/pointing.

Eccentricity of Loads (e)

(i) Floors/roof slab of normal span (< 30 × thickness of wall) having *full bearing* on external wall or a continuous slab on interior wall, the loading shall be considered to be *concentric*. If independent slabs of almost *equal span* (not differing by more than 15%) rest on an interior wall, the loading shall *also* be considered to be *concentric*.

(ii) If independent slabs of spans differing by more than 15% rest on an interior wall from both sides, the displacement of the point of application of each floor load shall be taken as *one-sixth of its bearing width* on the walls and the resultant eccentricity calculated therefrom.

(iii) If the span of the slab is more than 30 times its thickness, the eccentricity of the load is taken as $\frac{1}{6}$ th of the bearing width.

(iv) Floor/roof slab of normal span (< 30 × thickness of wall) not fully supported on *exterior* wall, the eccentricity of loads shall be $\frac{1}{12}$ th of the thickness of wall.

(v) When resultant eccentricity ratio of loading does not exceed $\frac{1}{24}$, compressive stress due to bending shall be ignored and only axial stress may be taken for design.

Taking *slenderness ratio* and *eccentricity of loads* into consideration, the *basic compressive stress* is modified by 'Stress Reduction Factor' (k_s) given in Table 7.5.

Table 7.5 : Stress Reduction Factor for Slenderness Ratio and Eccentricity

Slenderness Ratio	Eccentricity of loading divided by the thickness of the member					
	0	**1/24**	**1/12**	**1/6**	**1/4**	**1/3**
6	1.00	1.00	1.00	1.00	1.00	1.00
8	0.95	0.15	0.94	0.93	0.92	0.91
10	0.89	0.88	0.87	0.85	0.83	0.81
12	0.84	0.83	0.81	0.78	0.75	0.72
14	0.78	0.76	0.74	0.70	0.66	0.66
16	0.73	0.71	0.68	0.63	0.58	0.53
18	0.67	0.64	0.61	0.55	0.49	0.43
20	0.62	0.59	0.55	0.48	0.40	0.34
22	0.56	0.52	0.48	0.40	0.32	0.24
24	0.51	0.47	0.42	0.33	0.24	---
26	0.45	0.40	0.35	0.25	---	---
27	0.43	0.38	0.33	0.22	---	---

Note 1: Where, in special cases, the eccentricity of loading lies between 1/3 and 1/2 of the thickness of the member, the stress reduction factor should vary linearly between unity and 0.20 for slenderness ratio of 6 and 20 respectively.

Note 2: Slenderness ratio of a member for sections within 1/8 of the height of the member above or below a lateral support may be taken to be 6.

Example 7.2

Determine Eccentricity Ratio of wall A of the first floor of Example 7.3 (Figure 7.8) if there is an additional *udl* of 20 kN/m at an eccentricity of 30 mm from centre line of the wall.

Solution

Loads

Eccentric load 20 kN/m ≡ Axial load, 20 kN/m + a Moment,
M = 20 × 0.03 = 0.6 kNm/m

Load of parapet 1m high and 230 mm thick

\+

floor, roof and wall loads as calculated in Answers to SAQ 2 (v) = 54.569 kN/m

Total axial load, W = (54.569 + 20) = 74.569 kN/m

$$\text{Axial stress} = \frac{W}{A} = \frac{74.569}{1000 \times 220} = 3.39 \text{ N/mm}^2$$

$$e = \frac{M}{W} = \frac{0.6}{74.569} = 8.05 \text{ mm}$$

$$\textbf{Eccentricity Ratio} = \frac{8.05}{220} = \mathbf{0.037}\ \textbf{Ans}$$

Shape Modification Factor (k_p)

If the height to depth ratio of bricks *as laid* is more than 0.75, the basic compressive stress (σ_{ccb}) is *further* modified by Shape Modification Factor (k_p) given in Table 7.6

Table 7.6 : Shape Modification Factor for Masonry Units

Height to Width of Units (As laid)	Shape Modification Factor (kp) for Units having Crushing Strength in N/mm²			
	5.0	**7.5**	**10.0**	**15.0**
Upto 0.75	1.0	1.0	1.0	1.0
1.0	1.2	1.1	1.1	1.0
1.5	1.5	1.3	1.2	1.1
2.0 to 4.0	1.8	1.5	1.3	1.2

Area Reduction Factor

When sectional area of wall is less than 0.2 m², the *basic* permissible compressive stress is further modified by a factor called 'Area Reduction Factor' (k_a) where

$k_a = 0.7 + 1.5$ A

Thus the permissible compressive stress in brick masonry

$$\sigma_{cc} = \sigma_{ccb} k_s\, k_p\, k_a \qquad \ldots (7.1)$$

where SR = greater of $\frac{h}{t \times k_n}$ or $\frac{l}{t}$ used for deciding k_s.

Conditions for Modification of Permissible Compressive Stress

The permissible compressive stress value may be *increased* under certain *eccentric loading* conditions and in case of *concentrated loads* as given in Table 7.7

Table 7.7 : Increase in Permissible Compressive Stress

Conditions	Assumptions for Deisgn	% Increase of Permissible Compressive Stress (σ_{cc})
I *Eccentricity of loads (e)*		
(i) $\frac{1}{24} \leq e \leq \frac{1}{6}$		Increased by 25%
(ii) $e > \frac{1}{6}$	Area of section under tension shall be disregarded for load carrying capacity	
II Concentrated Loads on wall*		
(i) bearing on central strip *not* wider than half the thickness of the wall	Concentric	Increased by 50%
(ii) bearing on full thickness of wall	Concentric	Increased by 25%
(iii) bearing on central strip of wall wider than half the thickness but less than thickness of wall	Concentric	Increased by % age worked out by interpolation between values of increase in stresses in (i) and (ii) above
(iv) concentrated loads from lintel over an opening	Provided supporting area less than 3 times the bearing area	Increased by 50%

Permissible Tensile Stress

Generally it is assumed for design purposes that masonry is incapable of taking any tensile stress. However, in case of lateral loads normal to the plane of wall, which causes flexural tensile stress in panel, curtain, partition or free-standing walls, flexural tensile stress may be permitted as given in Table7.8

Permissible Shear Stress

Masonry walls built of M1 mortar or of higher grade and resisting horizontal forces in the plane of wall, permissible shear stress, calculated on the area of bed joint shall not exceed the value obtained by the formula given below to a maximum of 0.5 N/mm^2

$$f_s = 0.1 + f_d/6$$

where f_s = permissible shear stress in N/mm^2 and

f_d = compressive stress due to dead loads in N/mm^2

* A load is being taken to be concentrated when area of *supporting* wall equals or exceeds three times the bearing area. Maximum spread of a concentrated concentric load on a wall may be taken equal to b + 2t (b = width of bearing and t = thickness of wall) *or* stretch of wall supporting the load *or* c/c distance between loads, whichever is less.

Table 7.8 : Permissible Tensile Stress

Grade of Mortar	Direction of Bending and Direction of Tensile Stress on *bed-joint.*	Permissible Tensile Stress (N/mm^2)
Grade M1 or better mortar	For bending in the *vertical* direction where tension developed is *normal* to bed joint	0.07
	For bending in the *longitudinal* direction where tension developed is *parallel* to bed joints provided *crushing* strength of masonry *units* is not less than 10 N/mm^2	0.14
Grade M2 Mortar	For bending in the *vertical* direction where tension developed is *normal* to the bed joints	0.05
	For bending in *longitudinal* direction where tension developed is *parallel* to bed joints provided *crushing* strength of masonry *units* is *not* less than 7.5 N/mm^2	0.10

Note 1 No tensile stress is permitted in water or earth retaining structures.

Note 2 Allowable tensile stress in bending in the *vertical* direction may be increased to 0.1 N/mm^2 for M1 mortar and 0.07 N/mm^2 for M2 mortar in case of boundary walls/compound walls at the discretion of the designer.

Stability

A load bearing building upto 4 storey shall be considered to be stable if

(i) Height to width ratio in less than 2.

(ii) Cross (stiffening or shear) walls acting continuous from outer wall to outer wall or outer wall to load bearing inner wall of thicknesses and spacing given in Table 7.9 are provided.

7.2.2 Non-load Bearing Walls

A non-load bearing wall is designed to resist *only lateral loads*. Hence it is provided as an exterior wall to protect against weather and as an interior wall for partitioning purposes. Based on the uses mentioned above a non-load bearing wall may be called a panel wall or a curtain wall or a partition wall.

Panel walls are non-load bearing external walls in *framed* construction wholly supported on *each* storey and subjected to lateral loads only.

Curtain walls are non-load bearing walls supported by horizontal or vertical structural members, where necessary, and subjected to lateral loads only (Figure 7.4)

Partition walls are non-load bearing walls of one storey or part of a storey in height.

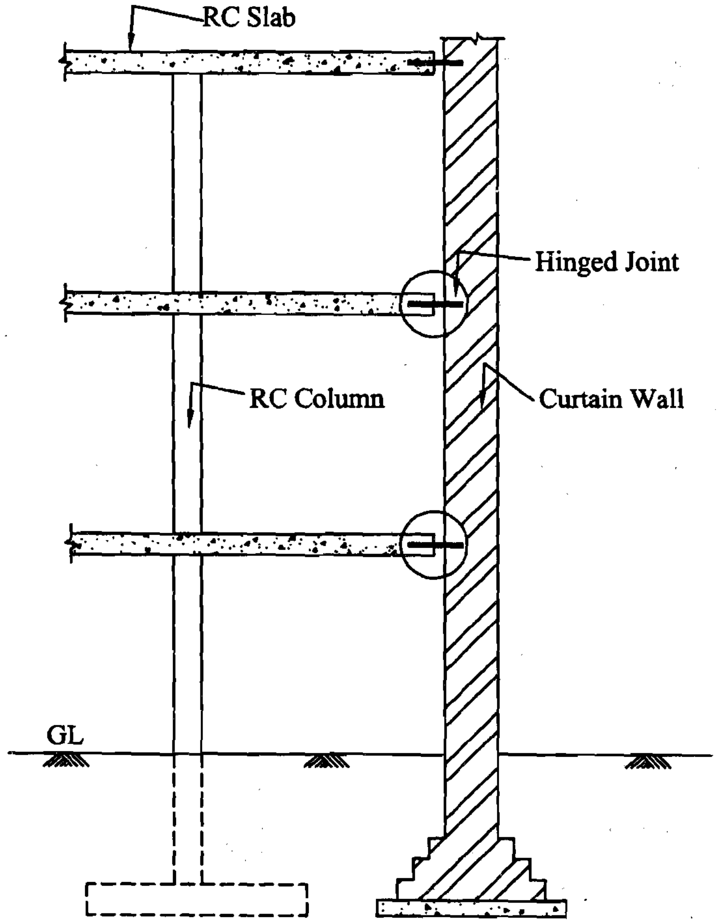


Figure 7.4 : Masonry Curtain Wall

Table 7.9 : Thickness and Spacing of Stiffening Walls

Thickness of Load Bearing wall to be Stiffened (mm)	Height of storey not to exceed (m)	STIFFENING WALL		
		Thickness (mm) not less than for		Maximum Spacing (m)
		1 to 3 storey	4 storeys	
100	3.2	100	200	4.5
200	3.2	100	200	6.0
300	3.4	100	200	8.0
300 above	5.0	100	200	8.0

Note 1 Storey height and maximum spacing as given are c/c dimension

Note 2 In case of halls exceeding 8 m in length, safety and adequency of lateral supports shall always be checked by structural analysis.

Note 3 A wall carrying gravity as well as lateral forces may be deemed to have been provided with adequate lateral support if the supporting walls (stiffening or cross walls) are capable of resisting reactions of all lateral forces plus 2.5% of the vertical load

Design of Non-load Bearing Walls

Design of Panel walls : Based on openings and support conditions of a wall, critical values of bending moments may be taken for design purposes from Table 7.10)

Table 7.10 : Bending Moments for Different Opening and Support Conditions

Conditions	Assumed support and span conditions	Bending Moments
Narrow and tall windows on either side of a panel	Simply suppoted, spanning in vertical direction	PH/8 (where H = c/c height and P = total hor. load on panel)
Long hor. windows between top support and the panel, the top edge of the panel is free	Supported on sides and at the bottom, spanning in horizontal direction (l)	For $\mu^* = 0.5$ Refer Table 7.11
No windows or 'hole-in-wall' type openings	Simply supported on all four edges	For $\mu = 0.5$ Refer Table 7.12

Table 7.11: Bending Moments in Laterally Loaded Panel Walls, Free at Top Edge and Supported on other Three Edges

$\frac{\text{Height of panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending moments	$\frac{PL}{25}$	$\frac{PL}{18}$	$\frac{PL}{14}$	$\frac{PL}{12}$	$\frac{PL}{11}$	$\frac{PL}{10.5}$	$\frac{PL}{10}$

Note For *H/L* ratio less than 0.30, the panel should be designed as a free-standing wall and for *H/L* ratio exceeding 1.75, it should be designed as a horizontally spanning member for a bending moment value of *PL*/8.

Table 7.12 : Bending Moments in Laterally Loaded Panel Walls Supported on all Four Edges

$\frac{\text{Height of panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending moments	$\frac{PL}{72}$	$\frac{PL}{36}$	$\frac{PL}{24}$	$\frac{PL}{18}$	$\frac{PL}{15}$	$\frac{PL}{13}$	$\frac{PL}{12}$

Note When *H/L* is less than 0.30, value of bending moment in the horizontal direction may be taken as nil and panel wali may be desinged for a bending moments value of *PH*/8 in the vertical direction; when *H/L* exceeds 1.75, panel may be assumed to be spanning in the horizontal direction and designed for bending moment of *PL*/8.

$$^*\mu = \frac{\text{Flexural strength of wall in the vertical direction}}{\text{Flexural strength of wall in the horizontal direction}}$$

Design of Curtain Walls

Curtain walls may be designed as panel walls taking *actual* support conditions into consideration.

Design of Partition Walls

Since these walls have to resist *smaller* lateral loads in comparison to other types, they may be apportioned emperically as given in Table 7.13

Table 7.13: Critical Dimensions of Walls for Different Support and Other Conditions

Support and other conditions	Dimension
I. *Walls with adequate lateral restraints at both ends but not at top*	
(a) panel of any height (H)	$L < 40t$
(b) panel of any length (L)	$H < 15\ t$
(c) $40t < L < 60t$	$(H + 2L) < 135\ t$
II. Walls with adequate lateral restraints at both ends and at the top	
(a) panel of any height (H)	$L < 40t$
(b) panel of any length (L)	$H < 30\ t$
(c) $40 < L < 60t$	$(L + 2H) < 200\ t$
III. When walls have adequate lateral restraints at top but not at ends	
(a) panel of any length	$H < 30\ t$

Note 1 In partition and in adjoining masonry, strength of bricks used should not be less than 3.5 N/mm^2 and grade of mortar not leaner than M2.

SAQ 1

(i) Define a load beaaring wall

(ii) What are the loading and other consideration made while designing a load bearing wall

(iii) What are the factors taken into consideration while determining the *basic compressive stress* in a wall

(iv) Define Slenderness Ratio (SR) of a wall. How SR of a wall is determined ?

(v) Design wall B of a two storeyed building of Example 7.3 Figure 7.8

(vi) Explain Area Reduction Factor

(vii) Specify under which considerations the permissible compressive stress is modified.

(viii) Enumerate and explain the considerations when tensile stress is permitted in Masonry.

(ix) Specify the permissible shear stress

(x) Explain the stability conditions for a builing made of masonry.

(xi) Write short notes on different types of Non-load Bearing Walls.

7.3 DESIGN CONSIDERATIONS FOR MASONRY COLUMNS, PIERS AND BUTTRESSES

Definition

A **column** is an isolated vertical member having width to thickness ratio not exceeding 4.

A **pier** is a thickened portion of a continuous wall provided at certain interval to increase stiffness of the wall or to carry concentrated loads.

A **buttress** is a pier projecting from either or both surfaces of a wall, decreasing in cross sectional area from base to top.

Design Considerations

The *slenderness ratio* for evaluation of permissible compressive stress in masonry for a coulmn, pier or buttress shall be *greater* of *effective height* to. respective effective thickness in the two principal directions. This ratio shall not exceed 12.

Effective height shall be taken as *actual height** for the direction it is laterally supported and as twice the actual height for the direction it is not laterally supported.

When there are openings in a wall and if its portion between the openings is, by definition, treated as a column, the effective height may be taken as given in Table 7.14

Table 7.14 : Effective Height of a Wall Portion Between Openings Treated as Column

Condition of Support	Effective Height
I. When a wall has full restraint at the top	
(i) for direction *perpendicular* to the plane of wall	0.75 H + 0.25 H H = distance between supports H_1 = Height of taller opening
(ii) for direction *paralled* to the plane of wall	H
II. When wall has partial restraint at the top	
(i) for direction *perpendicular* to plane of wall	H (when height neither opening exceeds 0.5 H) 2H (when height of any opening exceeds 0.5 H)
(ii) for direction *paralled* to the plane of wall	2H

The other design considerations are the same as those for walls.

SAQ 2

(i) Define Column, Pier and Buttress

(ii) Explain the consideration made for the design of column, pier or buttress

* Actual height is clear height between supports.

7.4 DESIGN OF FOOTINGS

Footings are provided below the ground level to transfer total load of super structure to the soil (Figure 7.5). The projection of any footing course shall be not more than half the depth of that course or 1/4th the brick length. The angle of dispersion of the load may be between 45^0 to 60^0 and the bearing capacity of the soil shall not be exceeded. There shall not be any tensile stress at the foundation level. A base course of lean concrete (1:3:6 or 1:4:8) shall be provided. The footing should be so designed that the upward reacton from soil should not be able to break or crack the concrete bed.

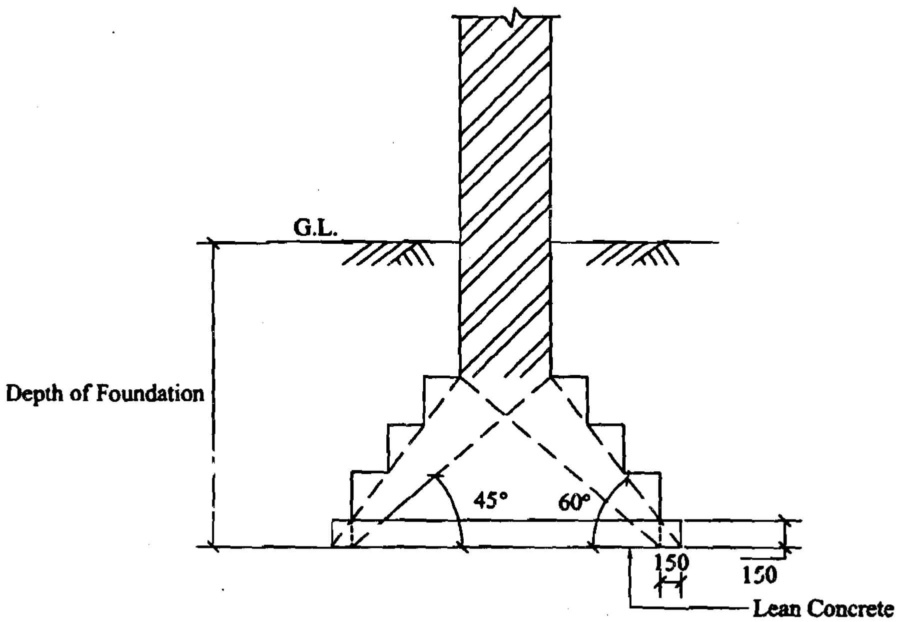


Figure 7.5 : Details of a Footing

SAQ 3

Explain with sketch different features taken into consideration for design of a footing.

7.5 DESIGN OF PIERS & ABUTMENTS FOR BRIDGES

Piers and abutments are substructures (supporting structures) in bridges and culverts ; whilst the former an intermediate support and the latter an end support retaining earth on their back. Brick or stone masonry piers and abutments of solid mass are constructed mainly for lighter loads from super structure.

7.5.1 Design Considerations for Solid Masonry Piers

Salient Features of Piers (Figure 7.6)

The *height* of a pier is fixed at 1 to 1.5 m above the Highest Flood Level (HFL) and is measured upto the support level of girder or springing point of an arch.

The *width of top* should be the bearing width on either side plus at least 150 mm.

Bligh has proposed a top width of '$\sqrt{span}$ 'where as Rankine has suggested a top width of $\frac{1}{6}$th to $\frac{1}{7}$th of span.

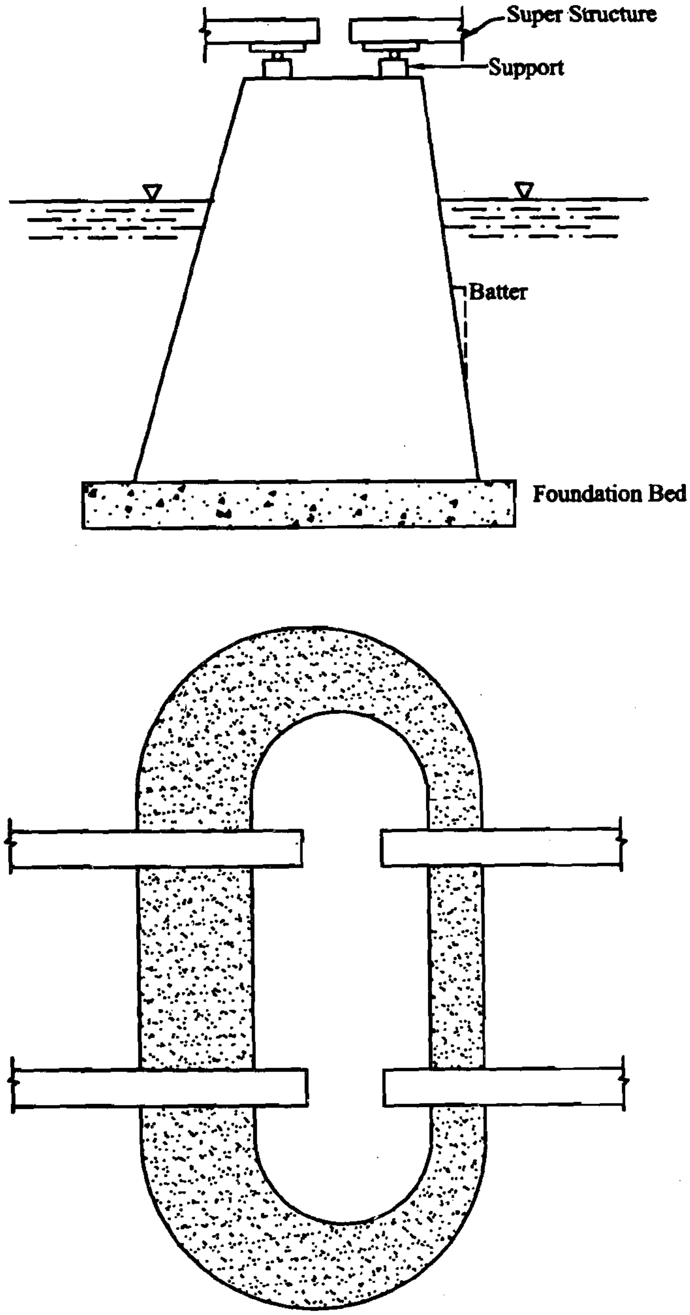


Figure 7.6 : Details of a Pier

The *width at the bottom* is generally $\frac{1}{3}$rd of its total height.

The *length* is, generally, one and half times the top width beyond the centre line of the girdı plus length of cut-water or ease-water*.

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* They are provided for *easy* passage of water. The *cut-water* is provided in up strea side where as *ease-water* at down stream side.

Design load for Piers

Design loads are :

(i) Dead Load ie self weight of pier

(ii) Reaction due to dead load of super structure and live load on it

(iii) Inpact load due to live loads and those due to floating bodies for upto 3 m of pier from top.

(iv) Lateral loads due to wind or seismic load, and

(v) Buoyancy effect of submerged part of the structure.

Other considerations

The pier must be safe against sliding and overturning. No tension shall be developed in any part of a pier and bearing capacity of soil shall not be exceeded.

7.5.3 Design considerations for Solid Masonry Abutments

Salient Features of Abuments (Figure 7.7)

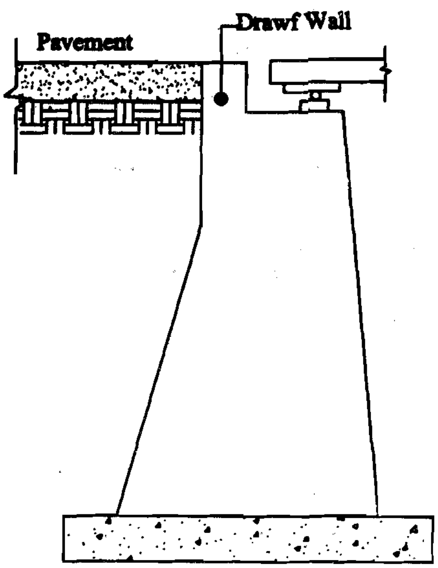

Figure 7.7 : Details of an Abutment

All the specifications required for design of piers are applicabe for the design of abutments except those mentioned below :

(i) The *top width* should be enough to accommodate the supports of super-structure and construction of dwarf wall to retain back-fill meterials of approach road, and

(ii) The *length* of abutment is at least equal to the width of the bridge.

Design Loads for Abutments

Design loads for abutments are the same as those for piers except that lateral pressure for the back fill may be considered in design. While taking critical lateral pressure, the pressure due to water on other side and vertical dead load reaction of the super structure may not be considered.

SAQ 4

Explain with sketches different design considerations and design loads for masonry piers and abutments for bridges.

Example 7.3

Design ground floor wall C for a double storey building (Figure 7.8) of the following specifications :

(i) Wall thickness = 230

(ii) clear height of walls in each floor = 3.00 m

(iii) floor consists of 125 mm thick RCC slab with 25 mm thick patent sotne

(iv) roof consists of 125 thick RCC slab with 125 mm lime concrete over it, and

(v) live loads on floor and roof shall be 3 kN/m^2 and 1.5 kN/m^2 respectively.

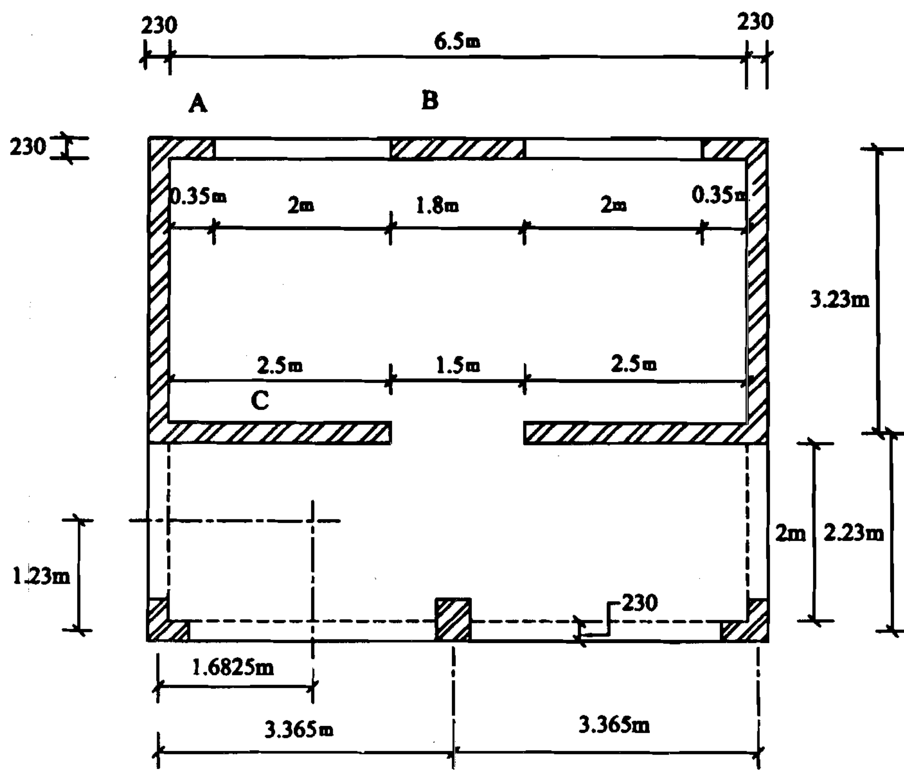

Figure 7.8 : Plan of the Building

Solution

Loads

As $\frac{l_y}{l_x} > 2$ for room and verandah, the loads from floor and roof will be assumed to be distributed on longer walls *only*. Hence load on wall C per meter width of wall will be from a strip of length $\frac{3.23 + 2.23}{2} = 2.73\text{m}$.

(i)	Loads from roof :		
	LC	$0.125 \times 1 \times 2.73 \times 20$	= 6.83 kN/m
	Slab	$0.125 \times 1 \times 2.73 \times 25$	= 8.53 kN/m
	LL	$1 \times 2.73 \times 1.5$	= 4.10 kN/m
(ii)	Loads from floor :		
	Patent stone	$0.025 \times 1 \times 2.73 \times 24$	= 1.64 kN/m
	Slab	$0.125 \times 1 \times 2.73 \times 25$	= 8.53 kN/m
	LL	$1 \times 2.73 \times 3$	= 8.19 kN/m
(iii)	Walls load above plinth :		
	Wall	$0.23 \times 1 \times 6 \times 20$	= 27.6 kN/m
	Total load		= 65.42 kN/m

Slenderness Ratio (SR)

Effective height of wall between plinth and floor, $h = 0.75\ H = 0.75 \times 3.063 = 2.3\text{m}$.

where H = height between plinth and centre line of floor

Effective length (l)

Total length of wall (l) = 6.5m + 0.23m = 6.73m

$$x\ (\text{Fig. 7.2}) > \frac{H}{8}\left(= \frac{3.063}{8}\text{m}\right)$$

$$y\ (\text{Fig.7.2}) > \frac{H}{6}\left(= \frac{3.063}{6}\text{m}\right)$$

$$\therefore\quad l = L = 6.73\text{m}$$

Effective Thickness (t)

$t = 220$

Stiffening Coefficient (k_n)

$s_p = 6.73\text{m}$

$w_p = 0.23\text{m}$

$$\therefore \frac{s_p}{w_p} = \frac{6.73}{0.23} = 29.26$$

$\therefore$ From Table 7.4, $k_n = 1$

SR from height consideration $= \frac{h}{tk_n} = \frac{2.3}{0.22 \times 1} = 10.45$

SR from length consideration $= \frac{l}{t} = \frac{6.73}{0.22} = 30.59$

Hence SR to be taken for design = 10.45 i.e. *lesser* of the above two values.

Eccentricity of loads

As wall C is loaded from both sides, the eccentricity of laod, $e = 0$

Stress Reduction Factor (k_s) : From Table 7.5 for

SR = 10.45 and $e = 0$, $k_s = 0.89 - \frac{(0.89 - 0.84)}{2} \times (10.45 - 10) = 0.88$

Actual compressive stress in masonry $= \frac{65.42 \times 10^3}{220 \times 1000} = 0.3 \text{ N/mm}^2$

with *Shape Modification Factor*, for $k_p = 1$

the *Design Basic Compressive Stress* for masonry

$$\sigma_{ccb} = \frac{0.3}{0.88 \times 1} = 0.34$$

Hence from Table 7.3 **provided masonry of bricks having crushing strength not less than 3.5 N/mm² and M2 (1:6) morlar** **Ans.**

Example 7.4

A free standing compound wall as shown in Figure 7.9 has to withstand a wind pressure of 300 N/m². Design the wall.

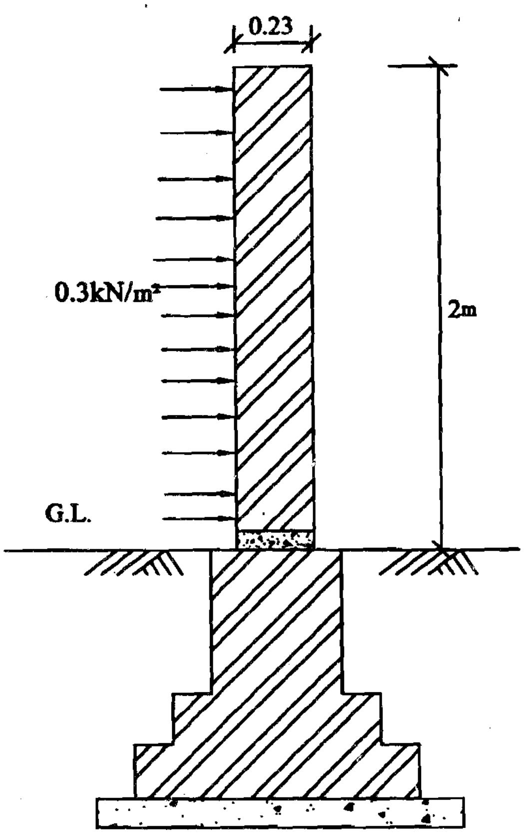

Figure 7.9 : A Free Standing Wall

Solution

Loads

Self wt. $= 0.23 \times 1 \times 2 \times 20$ $= 9.2$ kN/m

Bending moment at the base level, M $= 2 \times 1 \times 0.3$ $= 0.6$ kN/m

Stress at the base $f_C = \frac{P}{A} \pm \frac{M}{I} \cdot \frac{t}{2}$

$$= \frac{9.2 \times 10^3}{220 \times 1000} \pm \frac{0.6 \times 10^6}{\frac{1}{12} \times 1000 \times 220^3} \times \frac{220}{2}$$

$= 0.0418 \pm 0.0744$

$= 0.1162$ and 0.0326

Provided masonry of bricks of at least 3.5 N/mm² with M2 grade (1:6) mortar Ans.

Example 7.5

Design the footing of wall C of a *three* storey building (Figure 7.8) of the spcification given in example 7.3. Bearing capacity of soil = 120 kN/m².

Solution

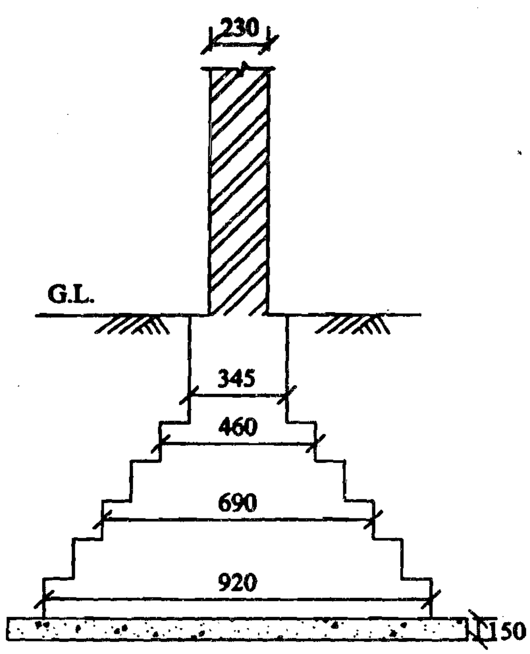

Figure 7.10 : Details of the Footing

Additional load on the footing than that given in Example 7.3 shall be the load of one more storey and self weight of the footing (ie wt. of masonry) below plinth level and that of lean concrete bed as shown in Figure 7.10.

Additional loads

Floor finish	(Refer Ex 7.3)	= 1.64 kN/m
slab	(Refer Ex 7.3)	= 8.53 kN/m
LL	(Refer Ex 7.3)	= 8.19 kN/m
Load of wall of one storey		
	0.23 × 3 × 20	= 13.58 kN/m
	Total additional load	= 31.94 kN/m
Load on wall C from two storey		
	(Refer Ex 7.3)	= 65.42 kN/m
	Total	= 97.36 kN/m
Self weight of footing		
(» 10% of above load)		= 9.74 kN/m
	Total	= 107.10 kN/m

Area of footing per meter width $= \frac{107.10}{120} = 0.8925 \text{ m}^2$

Provided a width of 920 mm as shwon in Figure 7.10.

Maximum pressure p_{max} on brick work in footing will be at *ground* level

$$p_{max} = \frac{97.36 \times 10^3}{345 \times 1000} = 0.282 \text{ N/mm}^2$$

Provided masonry of bricks having minimum strength equal to 3.5 N/mm² with M_2 (1:6) grade mortar **Ans.**

Example 7.6

Design the corner ground floor column shown in Fig. 7.8 of Example 7.3 for a *three* storey building.

Solution

Slenderss Ratio

Effective Height (h)

Since column is supported by plinth at plinth level and beams at floor level

effective height about x - x direction, h = 1.0 H = 3m

effective height about y - y direction, h = 1.0 H = 3m

Effective thickness (t)

Effective thicknes about x - x direction, t_x = average

$$\text{actual thickness} = \frac{330 + 220}{2} = 275$$

Effective thickness about y - y direction, t_x = average

$$\text{actual thickness} = \frac{330 + 220}{2} = 275$$

Slenderness Ratio about x - x axis $= (SR)_{xx} = \frac{h}{t_x} = \frac{3 \times 10^3}{275} = 10.91 < 12$

Slenderness Ratio about y - y axis $= (SR)_{yy} = \frac{h}{t_y} = \frac{3 \times 10^3}{275} = 10.91 < 12$

Hence **SR = 10.91**

Loads

Parapet	$= 0.23 \times (1.115 + 1.683) \times 1 \times 20$	= 12.87 kN
(assuming 1 m high parapet of 230 thickness)		
Roof Load		
Line concrete	$= 0.125 \times 1.23 \times 1.798 \times 20$	= 5.53 kN
Slab	$= 0.125 \times 1.23 \times 1.798 \times 25$	= 6.91 kN
LL	$= 1.23 \times 1.798 \times 1.5$	= 3.32 kN
Two floor Load		
Patent store	$= 2 \times 0.025 \times 1.23 \times 1.798 \times 24$	= 2.65 kN
Slab	$= 2 \times 0.125 \times 1.23 \times 1.798 \times 25$	= 13.82 kN
LL	$= 1.23 \times 1.798 \times 3$	= 6.63 kN
Self	$= 3 \times 3 \times (0.345 \times 0.23 + 0.115 \times 0.23) \times 20$	= 19.04 kN.
	Total	= 70.77 kN

$\therefore$ Compressive Stress, $f_{cc} = \frac{70.77 \times 10^3}{(345 + 115) \times 230} = 0.668 \text{ N/mm}^2$

for SR = 10.91 and e = 0

Stress Reduction Factor $k_s = 0.89 - \frac{(0.89 - 0.84)}{2} \times (10.91 - 10) = 0.867$

Shape Modification Factor, $k_p = 1$

(for brick size 220)

$\therefore$ Basic compr. stress reqd. $= \frac{f_{cc}}{k_s k_n k_p} = \frac{0.688}{0.867 \times 1 \times 1} = 0.77$

Hence provided masonry of bricks having minimum crushing strength of 10 N/mm² and cement mortar of type M2 (1:6) Ans

7.6 SUMMARY

Masonry is an assemblage of masonry units bonded together by mortar. Strength of masonry is basically a function of strengths of its constituents - masonry units and mortar. In this unit masonry, made of burnt clay bricks and cement mortars, which is mostly in use has been dealt with. Structural analysis and design of elements made of masonry depends on support and loading conditions, slenderness ratio, eccentricity of loads, laying of units, openings provided, stability of structure as a whole, etc. Safety and serviceability are the two main criteria for structural design.

7.7 ANSWERS TO SAQs

SAQ 1

(i) Refer text 7.2

(ii) Refer text 7.2

(iii) Refer text 7.2

(iv) Refer text 7.2

(v)

Loads

As $\frac{l_y}{l_x}$ >2 for the room, the load from floor and roof will be assumed to be distributed on longer walls only. Hence load on B per meter width of wall will be from a strip length of $\frac{3.23}{2} = 1.615\text{m}$

(i)	Load from roof :		
	Lime concrete	0.125 × 1 ×1.615 × 20	= 4.038 kN/m
	Slab	0.125 × 1 × 1.615 × 25	= 5.047 kN/m
	LL	1.0 × 1.615 × 1.5	= 2.423 kN/m
(ii)	Load from floor		
	Patent store	0.025 × 1 × 1.615 × 24	= 0.969 kN/m
	Slab	0.125 × 1 × 1.615 × 25	= 5.047 kN/m
	LL (3kN/m2)	1 ×1.615 × 3	= 4.845 kN/m
(iii)	Wall load above plinth		
	wall	0.23 × 1 × 6 × 20	= 27.600 kN/m
	Parapet (1m high and 230 other)		
		0.23 × 1 × 1 × 20	= 4.600 kN/m
	Total load		= 54.569 kN/m

Slenderress Ratio

Effective height of wall between plinth and floor = h = 0.75 H
= 0.75 × 3.063 = 2.3 m

where H = height between plinth and centre line of floor.

Effective length: As the length of wall of ends , x = 0.35 m

$$< \frac{H}{8}\left(= \frac{3.063}{8} = 0.383\right)$$

(vide Fig. 7.2 g) the effective length of wall = length of portion B of wall = 1.8 m > 4 × b_w* (= 4 × 0.23 = 0.92m)

Effective thickness (*t*)

t = 220

In this case SR will be determined by height only (Figure 7.2(g))

i.e. $SR = \frac{h}{tk_n} = \frac{2.30}{0.22 \times 1} = 10.45$

* The wall, between the openings is not a column by definition, as the length of wall is greater than 4t.

Eccentricity of Loads :

As the span of RC slab/roof, 3m<30 × thickness of wall (=30 × 0.23 = 6.9m) bears on the *external* wall B, e = 0

Stress Reduction Factor (k_s) : From Table7.3

for SR = 10.45 and e = 0, $k_s = 0.89 \frac{(.89-0.84)}{2} \times (10.45-10) = 0.88$

Shape Modification Factor : $\frac{\text{Height Units}}{\text{Width of a unit}} < 0.75$

∴ Shape Modification Facor = 1

Actual Compressive Stress on Masonry : At both ends of wall, reaction of lintel will be transferred to wall B

∴ Total Load on effective length L (1m) is for a length of wall = 1.8 m + half the lengths of opeings on both sides $= 1.8 + \frac{2 \times 2}{2} = 3.8\text{m}$

∴ $\frac{\text{Total load}}{\text{m}}$ = load per meter as calculated × $\frac{3.8}{1.8} = 54.569 \times \frac{3.8}{1.8}$
= 115.2 kN/m

Actual compressive Stress = $= \frac{115.20 \times 10^3}{220 \times 1000} = 0.524 \text{ N/mm}^2$

For shape Modification = 1, $k_s = 0.880$; $k_n = 1$

Basic compressive stress required = $\frac{\text{Actual Compressive Stress}}{\text{Shape Modification Factor} \times k_s \times k_n}$

$$= \frac{0.524}{1 \times 0.880 \times 1} = 0.6 \text{ N/mm}^2$$

Hence provided masonry having bricks of minimum crushing strength 7.5 N/mm² and mortar of type M1 (1:5) vide Table 2 & 3 Ans

(vi) Refer text 7.2

(vii) Refer text 7.2

(viii) Refer text 7.2

(ix) Refer text 7.2

(x) Refer text 7.2

(xi) Refer text 7.2

SAQ 2

(i) Refer text 7.3

(ii) Refer text 7.3

SAQ 4

Refer text 7.4

SAQ 5

Refer text 7.5